State of California | Business, Transportation and Housing Agency

Memorandum

To: MR. TOM POLLOCK, Chief
Office of Structure Design

Attention: Mr. Bob Anderson
Design Section 59-234

Date: September 18, 1992

File: 11-SD-56-0.02
11203 030111

El Camino Real UC
Bridge No. 57-1004R/L

#2

From: DEPARTMENT OF TRANSPORTATION
Division of New Technology, Materials & Research
Office of Engineering Geology - South

Subject: FOUNDATION RECOMMENDATIONS

INTRODUCTION

This office has completed a subsurface investigation at the proposed El Camino Real Undercrossing (Bridge No. 57-1004R/L) on Route 56, San Diego, CA. The investigation was a joint effort with District 11 Materials and consisted of drilling five rotary borings, two electric cone penetrometer (CPT) soundings, reviewing the site conditions and available records. Our investigation was based upon conversations with Design Section 10, the Foundation Plan received September 28, 1989 and the General Plan received August 19, 1991.

The proposed bridges will be approximately 186-foot long, single-span, prestressed, concrete-box structures. Approach fills will be placed at each abutment to a maximum height of approximately 41 feet. The bridge will be designed using 100 ton (compressive load) driven piles with tension capacities of 30 tons.

SUMMARY OF FINDINGS

Our findings are presented within this report and the Log of Test Borings (LTB). The LTB will be transmitted at a later date and is to be included in the contract plans. The layout sheet shows all borings drilled in the area, those borings not shown on the profile will be available through the Office of Geotechnical Engineering.

The site of the proposed El Camino Real UC is south of Carmel Valley Road and approximately 0.5 miles east of Route 5. The area of the proposed bridge is an undeveloped field which slopes gently to the south and west (1-3%) towards Carmel Valley Creek. Carmel Valley Creek is a tributary of the Soledad Valley estuary and is a sinuous, perennial stream that shows no incision below the active flood plain. The site is covered with native plants and grasses.

Exploratory borings reveal the bridge site is underlain by Holocene estuary and alluvial deposits (Power and others, 1982), which overlie Eocene bedrock. The estuary deposits (Qhe) are very loose to loose dark gray to brown to black, fossiliferrous silty sands to micaceous silts and clays deposited during landward intrusion of the sea. These silts and silty sands are interbedded with gray to light gray, slightly compact to very dense fluvial sands to silty sands (Qhfl). In the area of El Camino Real UC, the base of the fluvial deposits is well defined by a two to ten foot thick brown, dense to very dense, gravelly to cobbly sand. At elevations -23.8 and -29.4 feet in Borings 14L and

15L, respectively, these Holocene deposits overlie moderately-cemented, green to brown, Eocene mudstones and sandstones of the Delmar Formation (Td).

Ground Water

Ground water was measured between elevations 20.0 to 22.7 feet above sea level. The elevation of the ground water surface is highly dependent upon the seasonal rainfall. In general, from December to late April, ground water is at or near the ground surface.

Corrosivity

The following table lists the results from soil samples tested for corrosivity (California Test 643).

SAMPLE	pH	MINIMUM RESISTIVITY (Ohm-cm)	SULFATES (ppm)	CHLORIDES (ppm)	YEARS
B-14L @ 10-15	6.7	547	8220	148	20

The CALTRANS Corrosion Unit classifies sulfates in excess of 2,000 ppm and chlorides in excess of 500 ppm as corrosive. The number of years represents the length of time to perforate an 18 gage galvanized steel culvert. The limited testing indicates that the upper estuary deposits are corrosive to both steel and concrete. Soils below elevation +5 tested non corrosive.

Plasticity

A number of samples were selected and submitted for testing for Atterberg's Limits (California Test 204). The soils were found to be plastic and plotted outside the liquefiable range (Figure 1).

Sieve Analysis

A number of samples were selected and submitted for testing for grain-size distribution (California Test 202). The soils were found to be predominantly sandy silts and silty clays (Figure 2 and 3).

Seismicity

The Rose Canyon fault is mapped 5 miles west of the site (Reichle and others, 1990). The site is not within the Alquist-Priolo Special Study Zone (Hart, 1990). Mualchin & Jones (1991) proffer the following information for design of structures in the area:

Maximum Credible Earthquake Magnitude	7.0
Peak Horizontal Bedrock Acceleration	0.5 gravity

The depth to "rock-like" material (Vs greater than 2,500 feet per second) is 60 feet. The duration of strong-ground motion should be on the order of 15-20 seconds. The bridge site has not experienced ground shaking greater than 0.1 gravity in nearly 200 years (Reichle and others, 1990; Figure 2 & Table II).

Secondary Seismic Effects

Power and others (1982) performed a regional evaluation of liquefaction susceptibility in the San Diego Metropolitan area south of Carmel Valley. Their Table 1-1 indicates that the Holocene fluvial (Qhfl) and Holocene estuarine (Qhe) deposits, similar to those found in our borings, have a moderate to high susceptibility to liquefy during seismic events. They found that the estuarine and fluvial deposits have a mean blow count of 16 and recommended that site specific liquefaction studies be performed in areas where these deposits occur. Reichle and others (1990) hypothesized that Carmel Valley is not an area with high potential of experiencing ground failure due to liquefaction during an earthquake on the Silver Strand fault in Mission Bay.

Figure 4 illustrates the liquefaction susceptibility of the deposits underlying the El Camino Real Undercrossing. The analysis performed for this report utilized the method outlined by the National Research Council (1985), after Seed and Idriss (1982), and supplemented by Ishihara (*in press*). Figure 4 shows cyclic stress ratio versus normalized blow counts (adjusted for fines content) at 35' below the present ground surface for B-15L. The remaining samples taken in B-15L and B-14L were not plotted because these samples contained greater than 20% clay (0.005 mm) or blow counts greater than 30 and are therefore non-liquefiable.

Blow counts (abscissa) were determined using the method outlined by the National Research Council [NRC] (1985) and supplemented by Ishihara (*in press*). First, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586 incorporating the recommendations contained within NRC (1985; Tables 4-3 & 4-4). Secondly, the measured blow count (N) was normalized to one ton of overburden at 60% energy transfer or $(N_1)_{60}$ using the method outlined in NRC (1985). Third, sieve analysis (California Test 202) was performed to determine the influence of fines content (percentage of materials passing through the #200 sieve) as outlined by Ishihara (*in press*). Figures 2 and 3 show representative grain-size distributions for a number of samples. In B-14L and B-15L, all samples with blow counts less than 30 had a clay content (0.005 mm) greater than 20%. The one sample with blow counts less than 30 and a clay content less than 20% is B-15L @ 35'. Therefore, all of the sediments, with the exception of B-15L @ 35', are considered non-liquefiable because of the clay content(Seed and Idriss, 1982). For B-15L @ 35', $(N_1)_{60}$ was then converted to $(N_1)_{60} + \blacktriangle (N_1)_{60}$ using equation (12) from Ishihara (*in press*). This number $[(N_1)_{60} + \blacktriangle (N_1)_{60}]$ is plotted versus cyclic stress ratio (ordinate) to determine susceptibility to liquefaction for samples with a clay content less than 20 percent during a M=7.0 earthquake.

Cyclic stress ratio was determined by the methods presented in NRC (1985). Where a_{max} is the peak horizontal bedrock acceleration as determined from Mualchin & Jones (1991), r_d is the stress reduction factor of 1 at the surface to 0.9 at or below 35 feet. Total overburden and effective overburden were determined using saturated densities of 110 pcf and 130 pcf for estuary and fluvial deposits based upon samples taken in B-24L. These soil densities compare favorably to typical values of soil unit weight determined by Powers and others (1982).

Figure 4 indicates that, the sample taken at the 35 foot depth plots within the liquefiable zone. The sandy layer found at Boring 15L @ 35' (elevation -10) does not show in Boring E- 2L indicating that the sandy layer is discontinuous. Plotting the thickness of the non-liquefiable layer (H_1=35 feet) versus the thickness of the underlying liquefiable layer (H_2=±3 feet thick) on Figure 5 shows that the surficial manifestation of liquefaction is unlikely (Ishihara, 1985). Therefore, lateral spreading is not considered a hazard at the site.

In summary, soils in the area of the El Camino Real Undercrossing are not liquefiable with the exception of B-15L @ 35'. Liquefaction of this layer would likely not produce surface manifestations when liquefied, but may show settlement during a seismic event. To determine the amount of liquefaction induced settlement, the same data points are replotted versus volumetric strain (Figure 6), as recommended by Tokimatsu and Seed (1987). The volumetric strain is multiplied by thickness of the soil layer (3 feet from the CPT) to obtain a maximum dynamic settlement of 0.06 feet.

Settlement

Foundations: Calculations provided by the Office of Geotechnical Engineering indicate that dynamic settlement due to liquefaction can be as great as 1.21 feet near the El Camino Real UC and foundations should be designed against down drag forces along the pile.

Embankments: The Office of Geotechnical Engineering has recommended that stone columns be placed beneath the approach fills at the El Camino Real UC to mitigate the potential liquefaction hazard and support the fills; however, no construction sequence was provided in the memorandum dated August 17, 1992 or June 22, 1992. Calculations using the Hough Method estimated a settlement due to embankment surcharge of 2.1 feet at the embankments.

RECOMMENDATIONS

Additional Studies

The Office of Geotechnical Engineering should review and comment on the liquefaction susceptibility at the site and conduct additional studies as they deem necessary.

Seismic Hazards

Ground rupture is not a hazard at the site and, therefore, no special mitigative measures are required. Preliminary design of the bridge should be completed using a peak horizontal bedrock acceleration of 0.5 gravity and a depth to "rock-like" material of 60 feet. Final design should be based upon the site specific acceleration response study from the Office of Geotechnical Engineering.

Foundations

Foundations for the proposed bridge should be driven HP14 x 89 steel H-sections or 13 5/8 inch diameter, 1/2-inch thick wall pipe piles (open or closed end). The conical shaped tip is required for the pipe piles; the flat plate end is not an option. For both the open ended pipe and the H-section pile, tip protection is required. Concrete piles are not considered alternatives.

As required by Design Section 10, the minimum compressive capacity of the piles is 100 tons with a tension capacity of 30 tons. Pile capacities were calculated using the SPT method outlined by the FHWA and a minimum factor of safety of 2.0. Piles may be designed using the following table.

SUPPORT LOCATION	BOTTOM OF FOOTING ELEVATION	SPECIFIED TIP ELEVATION*	PILE LENGTH (FEET)	ULTIMATE COMPRESSIVE LOAD (TONS)	ULTIMATE TENSION LOAD FOR SEISMIC DESIGN (TONS)
Abut 1	+47-52	-29	76-81	200	90
Abut 2	+49-51	-30	79-81	200	90

*Probable Tip Elevations are estimated to be within 5 feet of specified tip.

Pile Load Tests

Dynamic and static (compressive and tension) pile load tests are planned for three locations within the interchange under the direction of the Office of Geotechnical Engineering. It is recommended that at least one of the load tests be performed in an area where the ground has been improved with stone columns and another in an area where no ground improvement has been done. Static load tests should be performed on the same day as driving to reduce the effects of soil set up. These tests should be performed prior to the driving of production piles for the bridge so that additional recommendations regarding the pile driving or construction sequence may be made if necessary. The location, specifications and layout for the pile load tests will be provided by the Office of Geotechnical Engineering.

Settlement

Foundations: Static and dynamic settlement of the foundations should be negligible because piles will be founded into the underlying bedrock. Piles founded into the bedrock will resist down drag (FHWA, 1986).

Embankments: After embankment fills have been placed to full height, an additional ten (10) foot high surcharge is recommended on the 100 feet of embankment closest the bridge. Settlement platforms should be installed and monitored by the Resident Engineer. A minimum settlement period of at least 120 days should be observed to allow for the approximately 2.1 feet of settlement; however, this settlement period may be accelerated by the installation of the stone columns. The settlement is complete when the rate of settlement is less than 1/4 inch over 10 consecutive days. The actual settlement period shall be determined by the engineer in the field.

Corrosion Protection

Most of the soil samples tested above the proposed pile tip elevations were all non corrosive; however, this does not preclude the possibility of corrosive layers unidentified by our testing. The heavier than normal steel piling will compensate for the limited areas of corrosive soils.

Approach Slabs

Seismic approach slabs will be required at both abutment locations.

Construction Specifications

The construction sequence should be as follows:

1. **Stone columns installed.**
2. **Embankments placed to full height with surcharge and settlement platforms installed.**
3. **Settlement period observed.**
4. **Piles driven.**

Predrilling may be required through the embankments fills to elevation +24. Hard driving (in excess on ENR 100 ton bearing) may be anticipated to attain specified tip elevation. The Special Provisions should state that if difficult driving is encountered, this office should be contacted prior to submission of pile driving alternatives (i.e.- jetting or predrilling) to the contractor.

The Special Provisions should state that the conical tip, or equivalent, is the only type of tip allowed for the closed end pipe piles. The Structure Representative should monitor initail pile installation efforts to evaluate the effect of the closed end on the driving. It is the option of the Structure Representative to remove the tip after consulting with this office.

Ground and surface water will effect construction. The contractor may be required to mitigate the effects of surface water in order to work. District 11 Environmental Planning should provide recommendations regarding restrictions on the work area.

Carmel Valley Road should be monitored for settlement during the pile driving operation.

REFERENCES CITED

Federal Highway Administration, 1986, Manual on Design and Construction of Driven Pile Foundations: U S Department of Transportation, Washington, D. C., p. 684.

Hart, E. W., 1990, Fault-rupture hazard zones in California: California Division of Mines & Geology Special Publication 42.

Ishihara, K., 1985, Stability of natural deposits during earthquakes; Proceedings of 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, California, v. 1, 321-376.

Ishihara, K., *in press*, Evaluation of Liquefaction potential and consequent deformations in sand fills.

Mualchin, L. & Jones, A. L., 1991, Peak acceleration from maximum credible earthquakes in California: California Division of Mines & Geology Open File Report 87 & Map Sheet 45.

National Research Council, 1985, Liquefaction of soils during earthquakes; National Academy Press, Washington, D.C.; Report No. PB86-163110, pp. 240.

Power, M. S., Dawson, A. W., Streiff, D. W., Perman, R. C. & Berger, V., 1982, Evaluation of liquefaction susceptibility in the San Diego, California urban area; v.1, U. S. Geological Survey Technical Report No. 14-08-0001-19110.

Reichle, M. S. and others, 1990, Planning Scenario for a Major Earthquake, San Diego-Tijuana Metropolitan Area; California Division of Mines and Geology Special Publication 100, pp. 181.

Seed, H. B. and Idriss, I. M., 1982, *Ground Motion and Soil Liquefaction during Earthquakes*; Earthquake Engineering Research Institute Monograph, v. 5, pp. 134.

Tokimatsu, K. and Seed, H. R., 1987, Evaluation of settlements in sands due to earthquake shaking: Journal of Geotechnical Engineering, v. 113, no. 8, 861-868.

Tokimatsu, K. and Yoshimi, Y., 1983, Empirical correlation of soil liquefaction based upon SPT N-value and fines content: Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, v. 23, n. 4, p. 56-74.

If you have any further questions, please do not hesitate to call (213) 620-3780 (ATSS-640-3780).

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Enclosures: Figures 1-6
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